Project Design Examples

PROJECT DESIGN EXAMPLES

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This part of *Volume IV* contains examples of how the rules, technical criteria and design aids presented in earlier parts might be used during the course of designing various types of surface water management systems. What is presented does not constitute additional rule criteria, and should not be used in lieu of the criteria or in a manner which is inconsistent with duly adopted rules.

Applicants are cautioned that these examples are not intended to provide guidance for all potential design aspects of surface water management systems. Specific project variables encountered – such as topography, soils, existing development, receiving body location, receiving body water quality classification, development density and wetland preserve areas – may dictate much more detailed or elaborate analyses.

AGRICULTURAL PROJECTS

The following two design examples represent two different agricultural projects. The main difference between the two is that one of them utilizes a major impoundment for water storage while the other utilizes a minor impoundment. (Impoundments are not for use exclusively in agricultural projects; large industrial projects, such as power plants, might also use either a major or a minor impoundment.)

Both design examples contain computer printouts representing hydrographic calculations. In order to reduce the volume of paper and still provide meaningful examples, portions of the printouts, which address stages and flows at various time steps, have been edited. The input data are presented along with the output summary in addition to a representative sample of the entire time step calculation.

The majority of the applications for permits to construct and operate surface water management systems serving agricultural land uses incorporate minor impoundments within their design. Although applications to construct major impoundments are not received as often, the potential damage that could occur if a major impoundment fails dictates that they be designed under criteria which are different from that which pertain to minor impoundments. Other than these differences, the following comments are intended to provide additional guidance on what usually needs to be considered in the design of a surface water management system serving an agricultural land use.

Soils and Vegetation

A review of the soil and vegetation within the area of proposed development will usually indicate areas, which are not feasible for farming because of environmental constraints and excessive drainage requirements. These areas will normally consist of poorly drained hydric soils and wetland vegetation. These areas are most often defined and delineated as wetlands pursuant to Chapter 62-340, F.A.C. and regulated pursuant to Section 4.0 of the *Basis of Review for Environmental Resource Permit Applications.* If properly designed, such areas can serve a dual purpose of preservation and storage, since onsite storage of water will probably be required in the final design.

One of the most important aspects to consider when incorporating wetlands into an above ground impoundment, is the control elevation of the impoundment in relation to the elevation of average or sustained wet season water levels (normal pool) of the wetland. Additionally, the peak stage and bleed down time are also critical to the successful preservation of the wetland habitat. The section in the Environmental Design Aids entitled *Protection of Wetland Hydroperiods* gives a basic overview of the factors to consider in this type of project design.

Floodplain Encroachment

The District considers two aspects of floodplain encroachment: storage reduction and flow interference. The 100-year storm event is the one that must be considered. Delineation of the 100-year floodplain, however, can be difficult in remote agricultural areas. In general, the storage volume between the 100-year flood elevation and the average wet season water table must be preserved in addition to maintaining a continuous flow cross section. The project detention area may partially serve this purpose but it may also be necessary for land outside of diked and farmed areas to remain undisturbed. One solution may include low farm dikes, which could be overtopped by 100-year flood stages since that amount of rainfall would destroy many crops anyway or might occur when no crops are planted.

Offsite Discharge

In most new farm areas of the District, where preconstructed works do not exist, it is necessary to accept upstream flows generated by a design storm event and pass them through or around the proposed project. In most cases, it will not be economical to attempt to mix the project runoff with the offsite flows as they enter the project site. The problems created by such designs include the backwater effects of the project on upstream lands, which may cause new upstream flooding.

Significant onsite storage area is therefore necessary because of the shallow storage depths which might be required. In flat areas where sheetflow of unknown direction predominates, the District usually requires that the toe of new farm dikes be kept away from property boundaries in order to allow water to move freely among the outside farm fields. The minimum setback distance is usually 50 feet, but may be more if it appears necessary. The construction of conveyance facilities may or may not be necessary in this setback area.

Onsite Storage

Onsite storage is necessary for quantity and quality management. It may also serve an additional purpose for irrigation supply. The District preference for storage areas is for separately contained areas fed by pumps, or gravity if the topography allows it. All discharge from the storage areas should be by gravity. Field protection is afforded under this scheme since fields can be pumped into the storage area. An internal emergency overflow structure is required in order to limit the head on the gravity structure which directs flows out of the water storage area. The external storage area levees should be more substantial than interior ones so that any failure would be internal rather than offsite.

Water Quality

The basic criteria applicable to projects are the detention of the first inch of runoff or the runoff from a 2.5 inch rainfall event, whichever is greater. The 1 inch criterion is the one that applies to agriculture. This first inch must be detained and allowed to be released within about 5 days. Actual retention, as opposed to detention, is usually difficult to accomplish because a guaranteed seepage system is necessary. Most farms are not in a location where this can be feasibly accomplished.

Design Example
for
An Agricultural Site

DESIGN EXAMPLE

FOR

AN AGRICULTURAL SITE

AGRICULTURAL SITE
(N.T.S.)

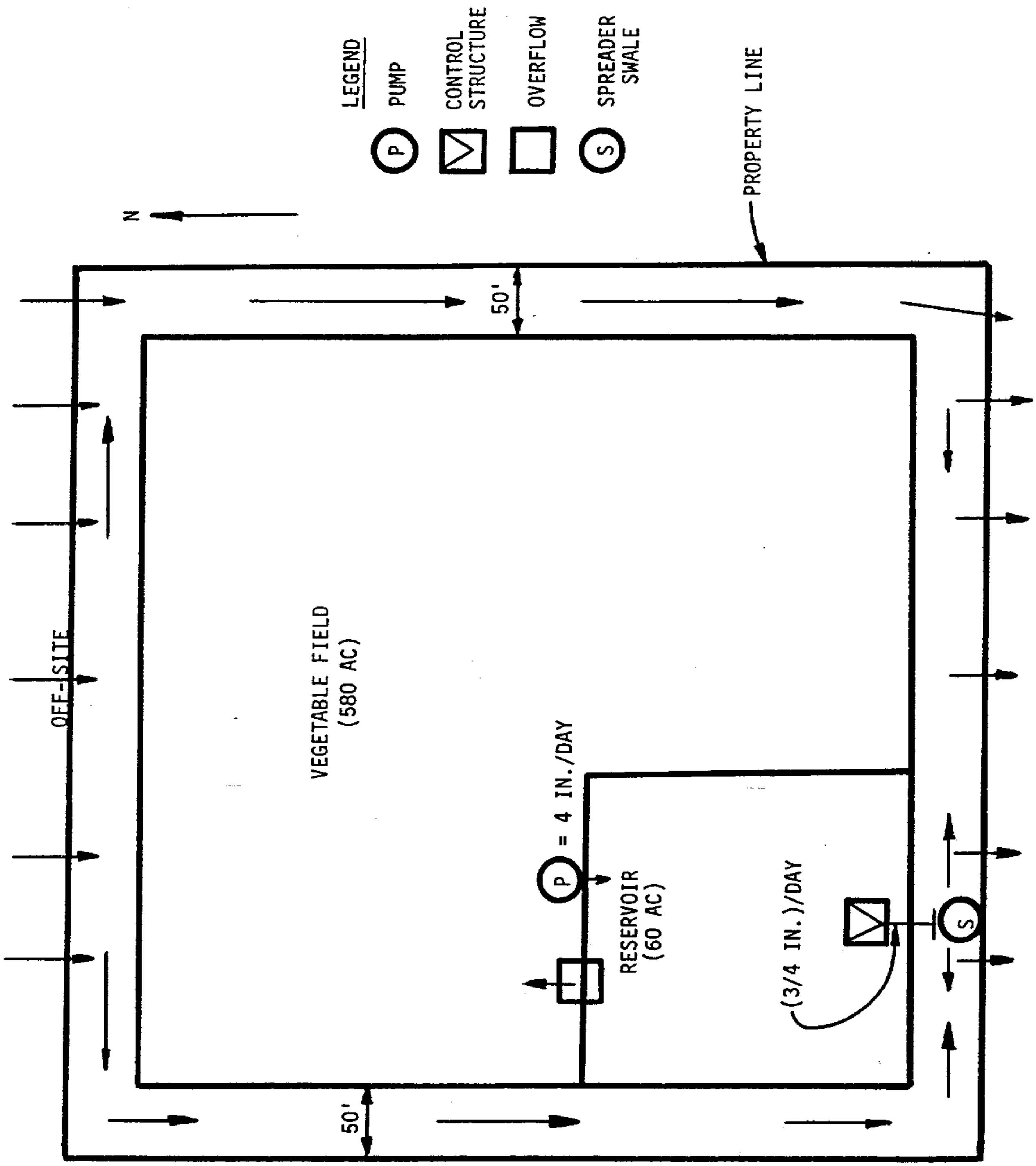


Figure XB-1

I. Given

A. Proposed Acreages

1. Total = 640 ac

2. Reservoir = 60 ac

3. Laterals = 40 ac

4. Vegetable Field = 540 ac

B. Elevations

1. Laterals extend from elevation 60' (bed) to 64' NGVD (banks).

2. Field extends from elevations 64' to 68' NGVD.

3. Field control elevation is 60' NGVD.

4. Reservoir control elevation is 65' NGVD, to maintain wetlands.

C. Off-site areas contribute sheetflow runoff to the site. The off-site runoff will be routed around the site via perimeter swales. (Toes of the berms are set back at least 50 feet from the property line; berms have a top width of 5 feet; have external sideslopes of 2.5H:1V, and interior sideslopes of 3H:1V for ease of maintenance.)

D. Discharge off-site will continue to be by sheetflow, to preserve natural conditions.

E. Depth to average wet season water table = 4.0 ft

F. A pumping rate of 4 in./day (97.6 cfs) from the field is proposed.

Pump capacity: 43,800 gpm = 97.6 cfs

Pump schedule	on	off
1 - 43800 gpm	61.5' NGVD	60.0' NGVD

G. Postdevelopment discharge allowed = 20 cfs = 3/4 inch per day.

NOTE: Discharge allowed can be determined two ways:

1. Predevelopment = postdevelopment

2. Receiving body discharge criteria if they have been established.

H. 25-year 3-day design storm rainfall = 7.0 in. x 1.359 = 9.5 in.

Treat the systems as two basins:

1. Basin 1 (field) routed into Basin 2

2. Basin 2 (reservoir) routed to receiving body.

II. Computations

A. Field

1. Compute Pervious/Impervious (P/I).

540 acre field

40 acre laterals (to be assumed impervious)

%I = (40 ac/580 ac) x 100% = 7%

%P = (540 ac/580 ac) x 100% = 93%.

2. Compute Soil Storage and SCS Curve Number.

a. Depth to average wet season water table = 4.0 ft.

b. From *Basis of Review*, 10.9 inches of moisture can be stored in the soil column beneath pervious areas.

c. Ground storage under pervious areas

= 10.9 in. x (1 ft/12 in.) x 540 ac

= 490.5 ac-ft.

d. Equivalent soil storage, S

= (490.5 ac-ft x (12 in./1 ft))/580 ac

= 10.2 in.

e. SCS curve number, CN

= 1000/(S + 10)

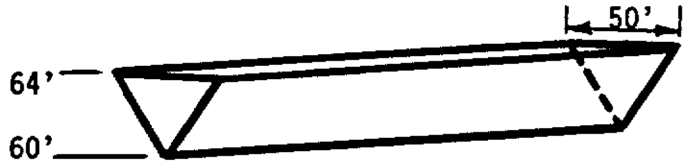
= 1000/(10.2 + 10)

= 50.

3. Compute Open Surface Stage versus Storage

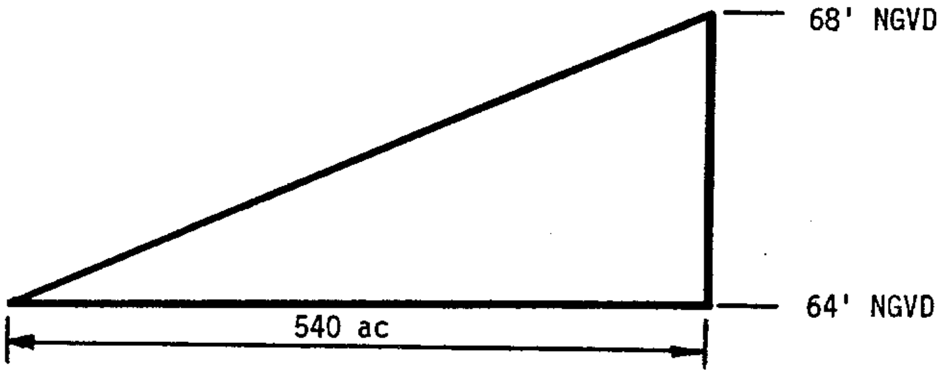
a. Laterals store between elevations 60' and 64' NGVD

Typical lateral cross-section



40 acres of laterals at top of bank

b. Developed field area stores linearly between elevation 64' and 68' NGVD.



	Storage		
Stage (ft,NGVD)	Laterals (ac-ft)	Site (ac-ft)	Total (ac-ft)
60	0	0	0
61	0.5(10ac)(1ft) = 5	0	5
62	0.5(20ac)(2ft) = 20	0	20
63	0.5(30ac)(3ft) = 45	0	45
64	0.5(40ac)(4ft) = 80	0	80
65	80 + 40ac(1ft) =120	0.5(540ac)((1x1ft)/4) = 67.5	188
66	80 + 40ac(2ft) =160	0.5(540ac)((2x2ft)/4) = 270	430
67	80 + 40ac(3ft) =200	0.5(540ac)((3x3ft)/4) = 607.5	808
68	80 + 40ac(4ft) =240	0.5(540ac)((4x4ft)/4) = 1080	1320

4. Flood Route the Design Storm

25-year 3-day event

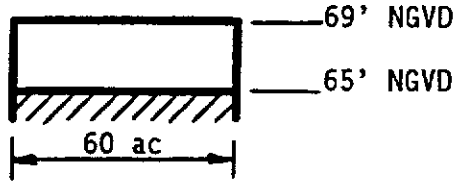
(See the pages titled "Example Grove," at end of this example)

Maximum stage in field = 63.6' NGVD.

B. Reservoir

1. S = 0.01 in. (District computer program will not accept zero.)

2. Since the reservoir will be used to mitigate for the loss of some wetlands, 1 foot of water will be maintained in parts of the reservoir at elevation 64.0' NGVD. Storage will then start at the control elevation, 65.0' NGVD.



Stage (ft,NGVD)	Storage (ac-ft)
65	0
66	60
67	120
68	180
69	240

3. Allowable Discharge

20 cfs was determined by pre- versus postdevelopment.

4. Weir crest elevation required.

a. First inch of runoff

b. Volume = 1 in. x (1 ft/12 in.) x 640 ac = 53.3 ac-ft. From the table in B.2., above, the weir crest should be set at elevation 65.9' NGVD, so that 53.3 ac-ft are detained.

5. Compute weir length

   a. Rainfall from the 25-year 3-day storm is 9 inches. Using the S value calculated for the field,

   $$\text{runoff} = Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = 3.2 \text{ in.}$$

   b. Volume of runoff

   = 3.2 in. x (1 ft/12 in.) x 640 ac

   = 171 ac-ft, or a reservoir stage of elevation 67.9' NGVD (table in B.2.), or 2 ft above the proposed weir crest.

   c. The maximum design head is 2.0 feet.

   $Q = 3.13LH^{1.5}$

   $L = 20 \text{ cfs}/(3.13 \times (2)^{1.5}) = 2.3$ feet

6. Water quality control device should be designed to discharge an amount (Q) equal to 1/2 inch in 24 hours.

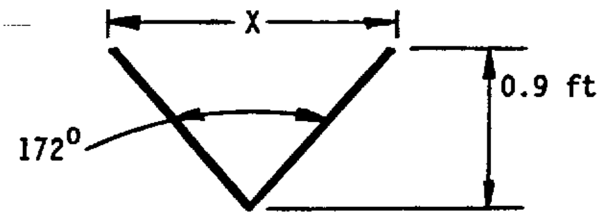
   53.3 ac-ft/2 = 26.7 ac-ft in 24 hours, or about 13 cfs

   $Q = 2.5 \times (\tan(\theta/2)) \times (H)^{2.5}$, or

   $\theta = 2 \times \tan^{-1}(0.492 \times Q/(H)^{2.5})$

   $\theta = 2 \times \tan^{-1}(0.492 \times 26.7/(0.9)^{2.5})$

   = 172 degrees



   $\tan(\theta/2) = (X/2)/0.9$

   $X = 2 \times (0.9 \text{ ft} \times \tan(172/2))$

   X = <u>25.7 ft</u> - will not fit into weir.

7. Try an orifice

$Q = 4.8 \times A \times (H)^{0.5}$

where Q = discharge, cfs

A = area of orifice, sq ft

H = head above notch centroid, ft

therefore, $Q = 20$ cfs*

$A = 0.5 \times b \times h$

assume $h = 2$ ft

Notch centroid will be 2/3 of h above notch invert, or 1.3 ft above elevation 65.0' NGVD, or at elevation 66.3' NGVD.

H = 67.9' NGVD - 66.3' NGVD

= 1.6 ft

$Q = 4.8 \times A \times H^{0.5}$

$= 4.8 \times (0.5 \times b \times h) \times H^{0.5}$

therefore,

$b = Q/(4.8 \times 0.5 \times h \times (H)^{0.5})$

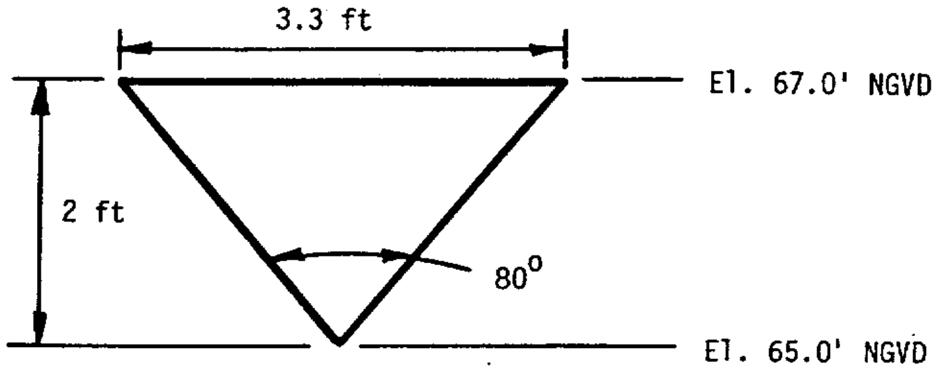
$= 20 \text{ cfs}/(4.8 \times 0.5 \times 2 \text{ ft} \times (1.6 \text{ ft})^{0.5})$

$= 20 \text{ cfs}/6.05$

$= 3.3$ ft.

*(NOTE: The orifice was designed using the more restrictive condition created by the allowable discharge criteria rather than the water quality rate.)

Therefore, the structure is 1-2.0 ft high by 3.3 ft wide triangular orifice with an invert at elevation 65.0' NGVD. Only an orifice is used since the allowable discharge is low and the lake size is preferred to be left unchanged, so a V-notch is impractical. The proposed control structure will be considerably smaller than what is required to meet the detention requirements for water quality. Therefore, discharge at elevation 67.0' NGVD will be considerably less than 0.5 inch in 24 hours.



Stage-discharge values for this control structure are given in the computer printouts at the end of this example. The structure should be properly baffled, to reduce the possibility of clogging. The outfall pipe should be large enough so that the peak design storm discharge is not limited by culvert control.

8. Check the routed design storm discharge.

   a. The design storm is the 25-year 3-day event.

   b. The one-day rainfall amount is given to be 7 inches.

   c. The three-day rainfall amount

      = (one-day amount) x 1.359

      = 7 in. x 1.359

      = <u>9.51 inches</u> of rainfall.

   d. The pages at the back of this example titled "Santa Barbara Program - Project Name: Example Grove.", are the results of calculations of pumping rainfall runoff into the reservoir. (Notice, on the final sheet, that the peak water surface elevation in the field was 63.57' NGVD.)

   e. The pages which begin with the one entitled "Santa Barbara Program-Project Name: Reservoir" are the routings of the pumped inflow, plus the rainfall on the reservoir.

      i. On the last sheet, the peak design storm discharge is listed as 20.6 cfs. Flows in excess of 20 cfs occur from hours 77.5 to 82.0 (not shown), and never exceed 20.6 cfs. This is deemed acceptable, since the allowed rate of 20 cfs was based on 3/4 in. per day.

ii. Also on the last sheet, the peak design storm elevation is listed as 68.01' NGVD. District recommended criteria are that, in the absence of a detailed dam structural safety analysis, the maximum above-grade stored depth of water shall be 4.0 feet. Since the average grade in the reservoir is 65' NGVD, which is just 3 feet less than the design storm peak,

the above-grade water depth is adequate.

iii. District recommended criteria on freeboard are 3 feet above the stored routed design storm.

Since the routed design storm peaks at elevation 68.0' NGVD,

the top of the reservoir berms should be no lower than elevation 71.0' NGVD.

9. Overflow Structure

a. Criteria

i. For gravity filled impoundments, no separate overflow structure is required, because the inflow structure will allow reservoir and field stages to be the same.

ii. The weir crest shall be set at the peak elevation of the routed 25-year 3-day storm.

iii. The weir crest length shall be adequate to pass the peak of:

the sum of the volume of the 100-year 3-day storm falling on the reservoir surface plus the inflow pump hydrograph for the same event, minus the routed discharge control structure outflow, with not more than 6 inches of head on the overflow structure weir crest.

iv. A simple culvert, or culverts, extending through the reservoir side, and with inverts at the design storm peak stage, are not acceptable, because of the danger of erosion of the reservoir dike.

v. There are at least two acceptable overflow structure designs.

I. A non-adjustable shaft spillway.

II. A properly designed and built non-adjustable sharp- or broad- crested weir in the reservoir dike.

vi. Sodded earthen berms are not acceptable as overflow structures.

vii. The inflow pump system shall be the same as that used for designing the control structure.

viii. Other proposed designs should be discussed with District staff as early in the design process as possible.

b. Design calculations

i. Treat the system as two basins: Basin 1 (field) routed into Basin 2 (reservoir).

ii. Basin 1 (field) pumps shall turn on at elevation 61.5' NGVD, and turn off at elevation 60.0' NGVD.

(Other methods of calculating the pumped flow hydrograph into the reservoir - for the purposes of computing the overflow weir length - may be submitted by the applicant for District staff consideration.)

iii. The stage-storage information for Basin 2 (reservoir) is unchanged from that previously calculated.

iv. After some preliminary calculations try a <u>70-foot long broad crested weir</u>.

v. The weir crest is set at the peak elevation of the 25-year 72-hour routed storm, in this case 68.0' NGVD, to the nearest tenth of a foot.

vi. The stage-discharge calculations for the overflow weir are based on the equation $Q = CLH^{1.5}$

where:

Q = discharge, cfs

C = weir coefficient, 3.00 for this broad-crested weir

L = weir length, in this case: 70.0 ft

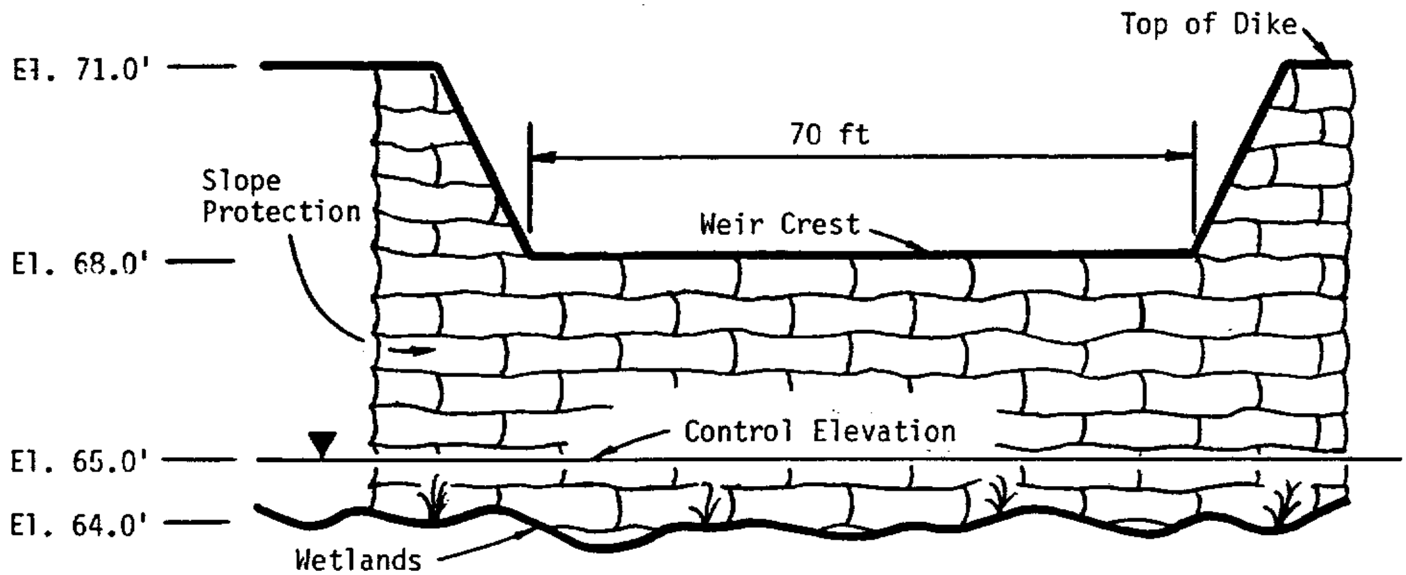
H = head on the weir, ft; which is not to exceed 0.5 ft

vii. The computer model utilized for analyzing the overflow structure contains a method for determining flows over the sharp-crested control structure weir which is slightly different from the method which was used in the design of the control structure. As a result, computed control structure flows will vary slightly between the two analyses, but the variance is negligible. Also, because the model calculation method will not work if the pump cut-off elevation is exactly equal to the lowest site storage elevation, the pump cut-off elevation is 60.03, instead of 60.0.

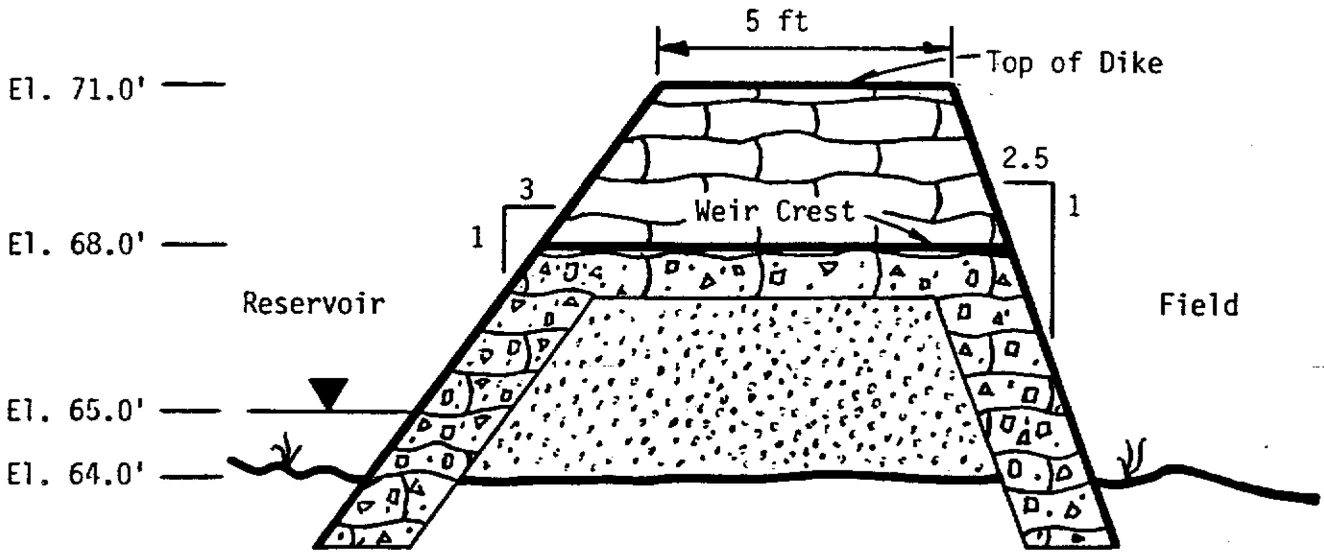
viii. The pages which begin "Santa Barbara - Project Name: Agricultural Example, Reservo" are selected from the hydrograph of the design event for the reservoir overflow structure. They show the results of routing the pumped inflow ("Structure 1") plus the 100-year 72-hour rainfall on the reservoir, out of the combination of the control ("Structure 2") and overflow ("Structure 3") structures.

The last page contains a summary of peak stage and discharges. Peak discharges occur at elevation 68.5' NGVD, which is 0.5 foot (6 inches) higher than the crest of the overflow structure. Therefore, <u>the trial overflow structure is adequate</u>.

A sketch of the proposed overflow structure is on the next page. A complete design would include, among other things, proper attention to erosion control around the overflow weir, and energy dissipation devices downstream of it.



Elevation View (N.T.S.)



Section View (N.T.S.)

OVERFLOW STRUCTURE

Figure XB-2

WEIR LENGTH 0 FT.
WEIR ELEVATION 70 FT. NGVD
WEIR COEFFICIENT 3.13
TYPE OF BLEEDER SLOT TRIANGLAR ORIFICE
SLOT INVERT ELEV. 65 FT. NGVD
ORIFICE HEIGHT 2 FT.
ORIFICE BASE WIDTH 3.3 FT.

STAGE	WEIR FLOW IN CFS WEIR	BLEEDER	TOTAL
65.00	0.00	0.00	0.00
65.10	0.00	0.01	0.01
65.20	0.00	0.04	0.04
65.30	0.00	0.11	0.11
65.40	0.00	0.23	0.23
65.50	0.00	0.41	0.41
65.60	0.00	0.64	0.64
65.70	0.00	0.94	0.94
65.80	0.00	1.31	1.31
65.90	0.00	1.76	1.76
66.00	0.00	2.29	2.29
66.10	0.00	2.91	2.91
66.20	0.00	3.62	3.62
66.30	0.00	4.42	4.42
66.40	0.00	5.32	5.32
66.50	0.00	6.32	6.32
66.60	0.00	7.43	7.43
66.70	0.00	8.64	8.64
66.80	0.00	9.97	9.97
66.90	0.00	11.41	11.41
67.00	0.00	12.98	12.98
67.10	0.00	13.92	13.92
67.20	0.00	14.80	14.80
67.30	0.00	15.63	15.63
67.40	0.00	16.41	16.41
67.50	0.00	17.17	17.17
67.60	0.00	17.89	17.89
67.70	0.00	18.58	18.58
67.80	0.00	19.25	19.25
67.90	0.00	19.89	19.89
68.00	0.00	20.52	20.52
68.10	0.00	21.12	21.12
68.20	0.00	21.71	21.71
68.30	0.00	22.29	22.29
68.40	0.00	22.85	22.85
68.50	0.00	23.39	23.39
68.60	0.00	23.93	23.93
68.70	0.00	24.45	24.45
68.80	0.00	24.96	24.96
68.90	0.00	25.46	25.46
69.00	0.00	25.95	25.95
69.10	0.00	26.43	26.43
69.20	0.00	26.91	26.91
69.30	0.00	27.37	27.37
69.40	0.00	27.83	27.83
69.50	0.00	28.28	28.28
69.60	0.00	28.72	28.72
69.70	0.00	29.16	29.16
69.80	0.00	29.59	29.59
69.90	0.00	30.01	30.01
70.00	0.00	30.43	30.43

S A N T A   B A R B A R A   P R O G R A M

PROJECT NAME . . . . . : EXAMPLE GROVE
REVIEWER . . . . :
PROJECT AREA . . . . . : 580.00 ACRES
GROUND STORAGE . . . . : 10.20 INCHES
TERMINATION DISCHARGE : 90.00 CFS
TIME OF CONCENTRATION : 2.00 HOURS
TIME STEP . . . . : .50 HOURS
DISTRIBUTION TYPE . . : SFWMD
RETURN FREQUENCY . . : 25.00 YEARS
RAINFALL DURATION . . : 3-DAY
24-HOUR RAINFALL . . : 7.00 INCHES
DISCHARGE HYDROGRAPH : GROVE1
REPORTING SEQUENCE . : INCREMENTAL

STAGE (FT)	STORAGE (AF)
60.00	.00
61.00	5.00
62.00	20.00
63.00	45.00
64.00	80.00
65.00	188.00

PUMP NO.	PUMP ON ELEVATION (FEET)	PUMP OFF ELEVATION (FEET)	PUMP DISCHARGE (GPM)	PUMP DESCRIPTION
1	61.50	60.00	43800.00	43800 GPM PUMP

				- - - - - - - R E S E R V O I R - - - - - -					
TIME (HR)	RAIN FALL (IN)	ACCUM. RUNOFF (IN)	BASIN DISCHGE (CFS)	ACCUM. INFLOW (AF)	VOLUME (AF)	ACCUM. OUTFLOW (AF)	INSTANT DISCHGE (CFS)	AVERAGE DISCHGE (CFS)	STAGE (FT)
.00	.00	.00	.0	.0	.0	.0	.0	.0	60.00
.50	.02	.00	.0	.0	.0	.0	.0	.0	60.00
1.00	.04	.00	.0	.0	.0	.0	.0	.0	60.00
1.50	.06	.00	.0	.0	.0	.0	.0	.0	60.00
2.00	.09	.00	.0	.0	.0	.0	.0	.0	60.00
2.50	.11	.00	.0	.0	.0	.0	.0	.0	60.00
3.00	.13	.00	.0	.0	.0	.0	.0	.0	60.00
3.50	.15	.00	.0	.0	.0	.0	.0	.0	60.00
4.00	.17	.00	.0	.0	.0	.0	.0	.0	60.00
4.50	.19	.00	.0	.0	.0	.0	.0	.0	60.00
5.00	.21	.00	.0	.0	.0	.0	.0	.0	60.00
5.50	.23	.00	.0	.0	.0	.0	.0	.0	60.00
6.00	.26	.00	.0	.0	.0	.0	.0	.0	60.00
6.50	.28	.00	.0	.0	.0	.0	.0	.0	60.00
7.00	.30	.00	.0	.0	.0	.0	.0	.0	60.00

					RESERVOIR				
	RAIN	ACCUM.	BASIN	ACCUM.		ACCUM.	INSTANT	AVERAGE	
TIME	FALL	RUNOFF	DISCHGE	INFLOW	VOLUME	OUTFLOW	DISCHGE	DISCHGE	STAGE
(HR)	(IN)	(IN)	(CFS)	(AF)	(AF)	(AF)	(CFS)	(CFS)	(FT)
73.50	9.51	3.16	19.4	149.5	36.6	112.9	97.6	97.6	62.65
74.00	9.51	3.16	15.1	150.2	33.3	116.9	97.6	97.6	62.52
74.50	9.51	3.16	11.8	150.8	29.8	121.0	97.6	97.6	62.38
75.00	9.51	3.16	9.1	151.2	26.2	125.0	97.6	97.6	62.24
75.50	9.51	3.16	7.1	151.6	22.5	129.1	97.6	97.6	62.09
76.00	9.51	3.16	5.5	151.8	18.7	133.1	97.6	97.6	61.94
76.50	9.51	3.16	4.3	152.0	14.9	137.1	97.6	97.6	61.65
77.00	9.51	3.16	3.3	152.2	11.0	141.2	97.6	97.6	61.40
77.50	9.51	3.16	2.6	152.3	7.1	145.2	97.6	97.6	61.14
78.00	9.51	3.16	2.0	152.4	3.2	149.2	97.6	97.6	60.88
78.50	9.51	3.16	1.6	152.5	-.7	153.2	.0	48.8	59.84

SUMMARY INFORMATION

MAXIMUM STAGE WAS 63.57 FEET AT 65.50 HOURS
MAXIMUM DISCHARGE WAS 97.6 CFS AT 59.50 HOURS

PUMPING HISTORY

PUMP 1 "ON " AT 59.50 HOURS
PUMP 1 "OFF" AT 78.50 HOURS

S A N T A   B A R B A R A   P R O G R A M

PROJECT NAME . . . . . : RESERVOIR
REVIEWER . . . . :
PROJECT AREA . . . . . : 60.00 ACRES
GROUND STORAGE . . . . : .01 INCHES
TERMINATION DISCHARGE : 10.00 CFS
TIME OF CONCENTRATION : .50 HOURS
TIME STEP . . . . : .50 HOURS
DISTRIBUTION TYPE . . : SFWMD
RETURN FREQUENCY . . : 25.00 YEARS
RAINFALL DURATION . . : 3-DAY
24-HOUR RAINFALL . . : 7.00 INCHES
INFLOW HYDROGRAPH . . : GROVE1 LAGGED BY .00 HOURS
REPORTING SEQUENCE . : INCREMENTAL

STAGE (FT)	STORAGE (AF)	DISCHARGE (CFS)
65.00	.00	.00
66.00	60.00	2.30
67.00	120.00	13.00
68.00	180.00	20.50
69.00	240.00	26.00

					- - - - - - - R E S E R V O I R - - - - - -				
TIME (HR)	RAIN FALL (IN)	ACCUM. RUNOFF (IN)	BASIN DISCHGE (CFS)	ACCUM. INFLOW (AF)	VOLUME (AF)	ACCUM. OUTFLOW (AF)	INSTANT DISCHGE (CFS)	AVERAGE DISCHGE (CFS)	STAGE (FT)
.00	.00	.00	.0	.0	.0	.0	.0	.0	65.00
.50	.02	.01	.5	.0	.0	.0	.0	.0	65.00
1.00	.04	.03	1.5	.1	.1	.0	.0	.0	65.00
1.50	.06	.05	2.1	.1	.1	.0	.0	.0	65.00
2.00	.09	.07	2.4	.2	.2	.0	.0	.0	65.00
2.50	.11	.10	2.5	.3	.3	.0	.0	.0	65.00
3.00	.13	.12	2.5	.4	.4	.0	.0	.0	65.01
3.50	.15	.14	2.6	.5	.5	.0	.0	.0	65.01
4.00	.17	.16	2.6	.6	.6	.0	.0	.0	65.01
4.50	.19	.18	2.6	.7	.7	.0	.0	.0	65.01
5.00	.21	.20	2.6	.8	.8	.0	.0	.0	65.01
5.50	.23	.22	2.6	1.0	.9	.1	.0	.0	65.01
6.00	.26	.24	2.6	1.1	1.1	.0	.0	.0	65.02
6.50	.28	.27	2.6	1.2	1.2	.0	.0	.0	65.02
7.00	.30	.29	2.6	1.3	1.3	.0	.0	.0	65.02
7.50	.32	.31	2.6	1.4	1.4	.0	.1	.0	65.02
8.00	.34	.33	2.6	1.5	1.5	.0	.1	.1	65.02
8.50	.36	.35	2.6	1.6	1.6	.0	.1	.1	65.03
9.00	.38	.37	2.6	1.7	1.7	.0	.1	.1	65.03
9.50	.40	.39	2.6	1.8	1.8	.0	.1	.1	65.03

					- - - - - -	R E S E R V O I R	- - - - -		
	RAIN	ACCUM.	BASIN	ACCUM.		ACCUM.	INSTANT	AVERAGE	
TIME	FALL	RUNOFF	DISCHGE	INFLOW	VOLUME	OUTFLOW	DISCHGE	DISCHGE	STAGE
(HR)	(IN)	(IN)	(CFS)	(AF)	(AF)	(AF)	(CFS)	(CFS)	(FT)

SUMMARY INFORMATION

MAXIMUM STAGE WAS 68.01 FEET AT 79.00 HOURS
MAXIMUM DISCHARGE WAS 20.6 CFS AT 79.00 HOURS

S A N T A   B A R B A R A

PROJECT NAME . . . . : AGRICULTURAL EXAMPLE, RESERVO
REVIEWER . . . . : YALN
PROJECT NUMBER - 1

THE ROUTING IS COMPLETE WHEN THE DISCHARGE FOR BASIN 2 IS REDUCED TO 10.00CFS

********************** BASIN 1 *****************

AREA - 580.00 ACRES   GROUND STORAGE - 10.20 INCHES

TIME STEP - 2.00 HOURS   TIME OF CONCENTRATION - 2.00 HOURS

RETURN FREQUENCY - 100.00 YEARS   RAINFALL DISTRIBUTION - 3-DAY   24-HOUR RAINFALL - 9.00 INCHES

STAGE (FT)	STORAGE (AF)
60.00	.00
61.00	5.00
62.00	20.00
63.00	45.00
64.00	80.00
65.00	188.00
66.00	430.00

********************** BASIN 2 *****************

AREA - 60.00 ACRES   GROUND STORAGE - .01 INCHES

TIME STEP - 2.00 HOURS   TIME OF CONCENTRATION - .50 HOURS

RETURN FREQUENCY - 100.00 YEARS   RAINFALL DISTRIBUTION - 3-DAY   24-HOUR RAINFALL - 9.00 INCHES

STAGE (FT)	STORAGE (AF)
65.00	.00
66.00	60.00
67.00	120.00
68.00	180.00
69.00	240.00
70.00	300.00

DISCHARGE STRUCTURE INFORMATION

STRUCT NO.	PIPE SLOPE (%)	DIAMETER (FT)	ROUGHNESS	LENGTH (FT)	WEIR WEIR CREST TYPE/ELEV	WEIR LENGTH	HEAD INVERT ELEVATION	TAIL INVERT ELEVATION	
1	SEE PUMP TABLE								
2									BASIN 2 TO BASIN 3
3					BROAD/ 68.0	70.00			BASIN 2 TO BASIN 1

BLEEDER INFORMATION

STRUCT NO.	BLEEDER TYPE	ORIFICE DIAMETER OR WIDTH (FT)	ORIFICE INVERT ELEVATION (FT-NGVD)	AREA (FT2)	V-NOTCH ANGLE (DEG)	V-NOTCH INVERT ELEVATION (FT-NGVD)	TOP ELEVATION (FT-NGVD)
1	NO BLEEDER INCLUDED IN STRUCTURE						
2	V-NOTCH	NA	NA	NA	80.0	65.0	67.0
3	NO BLEEDER INCLUDED IN STRUCTURE						

PUMP TABLE

PUMP NO.	PUMP ON ELEVATION (FEET)	PUMP OFF ELEVATION (FEET)	PUMP DISCHARGE (GPM)	DISCHARGES FROM BASIN	TO BASIN
1	61.50	60.03	43800.00	1	2

OFFSITE RECEIVING WATER

TIME(HR)	STAGE(FT-NGVD)
.00	60.00
1000.00	60.00

SUMMARY REPORT

TIME (HR)	PIPE NO	BASIN NO	CUMULATIVE RAINFALL (INCHES)	CUMULATIVE RUNOFF (INCHES)	INSTANT. RUNOFF (CFS)	RUNOFF HYDROGRAPH (CFS)	DISCHARGE (CFS)	INSTANT. STAGE (FT)	FROM	TO	STRUCTURE CONTROL
.0	1	1	.0	.0	.0	.0	.00	60.00	1	2	
	2	2	.0	.0	.0	.1	.00	65.00	2	3	
	3	2	.0	.0	.0	.0	.00	65.00	2	1	
2.0	1	1	.1	.0	.0	.0	.00	60.00	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	.1	.1	3.3	3.2	.00	65.01	2	3	BLEEDER
	3	2	.1	.1	3.3	3.2	.00	65.01	2	1	NO FLOW
4.0	1	1	.2	.0	.0	.0	.00	60.00	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	.2	.2	3.3	3.3	.00	65.01	2	3	BLEEDER
	3	2	.2	.2	3.3	3.3	.00	65.01	2	1	NO FLOW
6.0	1	1	.3	.0	.0	.0	.00	60.00	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	.3	.3	3.3	3.3	.00	65.02	2	3	BLEEDER
	3	2	.3	.3	3.3	3.3	.00	65.02	2	1	NO FLOW
8.0	1	1	.4	.0	.0	.0	.00	60.00	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	.4	.4	3.3	3.3	.00	65.03	2	3	BLEEDER
	3	2	.4	.4	3.3	3.3	.00	65.03	2	1	NO FLOW
10.0	1	1	.5	.0	.0	.0	.00	60.00	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	.5	.5	3.3	3.3	.00	65.04	2	3	BLEEDER
	3	2	.5	.5	3.3	3.3	.00	65.04	2	1	NO FLOW
12.0	1	1	.7	.0	.0	.0	.00	60.00	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	.7	.6	3.3	3.3	.00	65.05	2	3	BLEEDER
	3	2	.7	.6	3.3	3.3	.00	65.05	2	1	NO FLOW
14.0	1	1	.8	.0	.0	.0	.00	60.00	1	2	NO FLOW

TIME (HR)	PIPE NO	BASIN NO	CUMULATIVE RAINFALL (INCHES)	CUMULATIVE RUNOFF (INCHES)	INSTANT. RUNOFF (CFS)	RUNOFF HYDROGRAPH (CFS)	DISCHARGE (CFS)	INSTANT. STAGE (FT)	FROM	TO	STRUCTURE CONTROL
	2	2	12.2	12.2	.0	.0	11.70	66.99	2	3	BLEEDER
	3	2	12.2	12.2	.0	.0	.00	66.99	2	1	NO FLOW
174.0	1	1	12.2	5.1	.0	.0	.00	61.09	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	12.2	12.2	.0	.0	11.24	66.96	2	3	BLEEDER
	3	2	12.2	12.2	.0	.0	.00	66.96	2	1	NO FLOW
176.0	1	1	12.2	5.1	.0	.0	.00	61.09	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	12.2	12.2	.0	.0	10.81	66.93	2	3	BLEEDER
	3	2	12.2	12.2	.0	.0	.00	66.93	2	1	NO FLOW
178.0	1	1	12.2	5.1	.0	.0	.00	61.09	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	12.2	12.2	.0	.0	10.40	66.90	2	3	BLEEDER
	3	2	12.2	12.2	.0	.0	.00	66.90	2	1	NO FLOW
180.0	1	1	12.2	5.1	.0	.0	.00	61.09	1	2	NO FLOW
OFFSITE STAGE IS 60.00 FT.NGVD											
	2	2	12.2	12.2	.0	.0	10.02	66.87	2	3	BLEEDER
	3	2	12.2	12.2	.0	.0	.00	66.87	2	1	NO FLOW

STRUCTURE NO.	PEAK DISCHARGE (CFS)	TIME OF OPEAK	PEAK STAGE (FT-NGVD)	TIME OF HPEAK
1	97.6	54.7	64.4	67.2
2	23.7	115.6	68.5	115.6
3	73.9	115.6	68.5	115.6

Design Example
for
A Major Impoundment

DESIGN EXAMPLE

FOR

A MAJOR IMPOUNDMENT

AGRICULTURAL SITE WITH MAJOR IMPOUNDMENT

(N.T.S.)

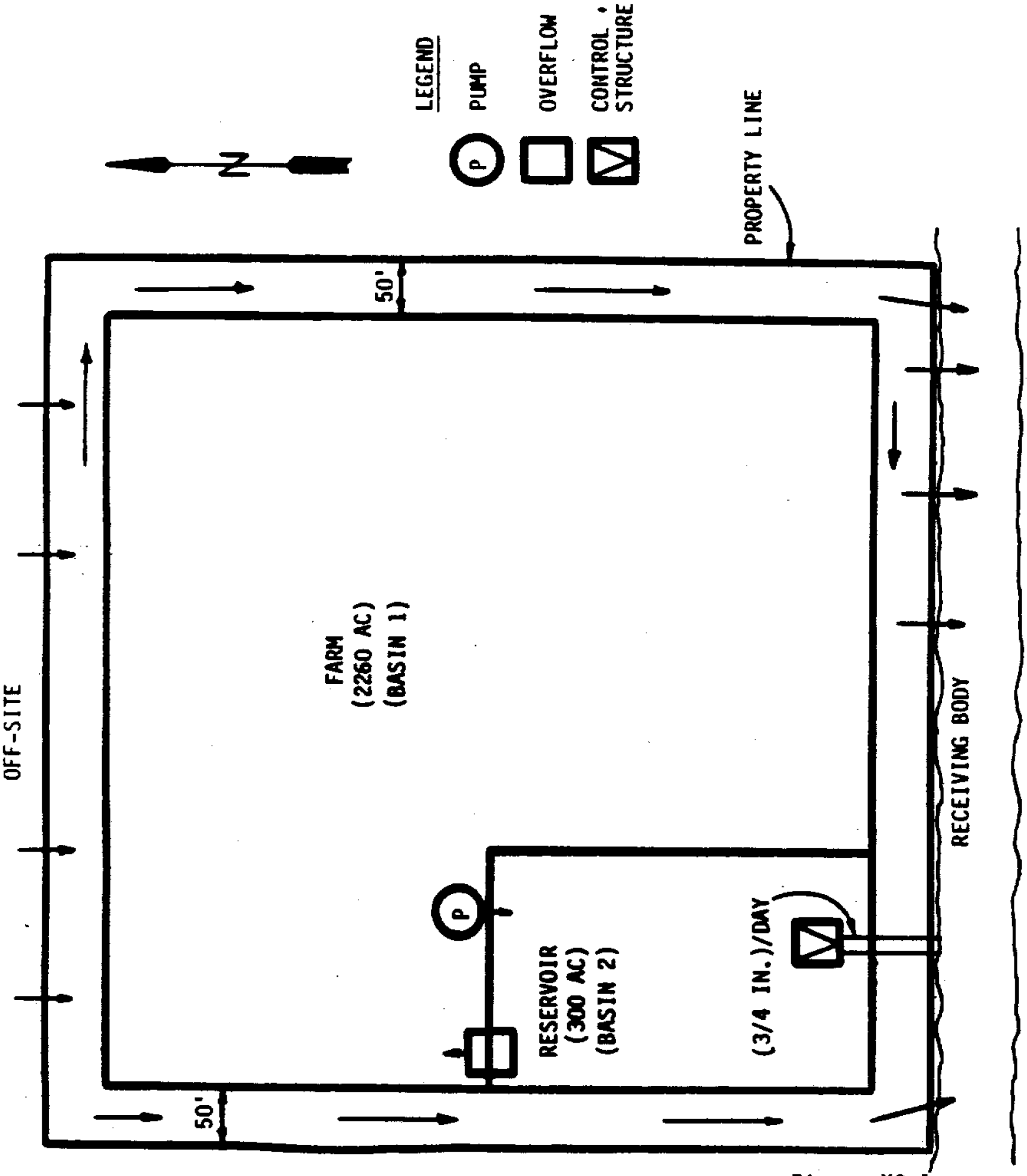


Figure XC-1

I. Given

A. Proposed Land Use

1. Total = 2560 ac

2. Reservoir = 300 ac

3. Lateral Canals = 60 ac

4. Sod Farm = 2200 ac

B. A large mobile home park is immediately adjacent to the west boundary of the sod farm.

C. Elevations

1. Laterals extend from elevation 18' (bed) to 22' NGVD (banks).

2. Developed farm site grading varies from elevation 22' to 25' NGVD.

3. Farm control elevation is 20' NGVD.

4. Reservoir control elevation is 23' NGVD to maintain wetlands.

D. Off-site areas contribute sheetflow runoff to the site. The off-site runoff will be routed around the site via perimeter swales. Discharge off-site will continue to be sheetflow, to simulate pre-development natural conditions. (Toes of the perimeter berms are set back at least 50 feet from the property line; berms have a top width of 5 feet; have external sideslopes of 2.5H:1V, and interior sideslopes of 3H:1V for ease of maintenance.)

E. Farm discharge will be routed through a control structure, culvert, and spreader swale to the adjacent receiving body.

F. Depth to average wet season water table is 2.0 ft.

G. A pumping rate of 4 in./day (380 cfs) from the farm to the detention area is proposed.

Pump capacity: 4 @ 42,636 gpm = 95 cfs; or 170,544 gpm = 380 cfs
Pump schedule: On at 21.5' NGVD, off at 20.0' NGVD

H. Post development discharge allowed = 81 cfs = 3/4 in. per day.

NOTE: Discharge allowed can be determined two ways:

1. Predevelopment = postdevelopment

2. Receiving body discharge criteria if they have been established.

I. 25-year 3-day design storm rainfall = 8.0 in. x 1.359 = 10.9 in.

Treat the systems as two basins:

1. Basin 1 (farm) routed into Basin 2 (reservoir).

2. Basin 2 (reservoir) routed to receiving body.

II. Computations

A. Farm

1. Compute Pervious/Impervious (P/I)

2200 ac farm (pervious)
60 ac laterals (assumed impervious for soil storage calculations)

%I = (60 ac/2260 ac) x 100 = 2.7%
%P = (2200 ac/2260 ac) x 100 = 97.3%.

2. Compute Soil Storage and SCS Curve Number

a. Depth to average wet season water table = 2.0 ft

b. From *Basis of Review*, 2.5 inches of moisture can be stored in the soil column beneath pervious areas.

c. Ground storage under pervious areas
= 2.5 in. x (1 ft/12 in.) x 2200 ac
= 458 ac-ft.

d. Equivalent soil storage, S

= 458 ac-ft x ((12 in./1 ft)/2260 ac)
= 2.4 in.

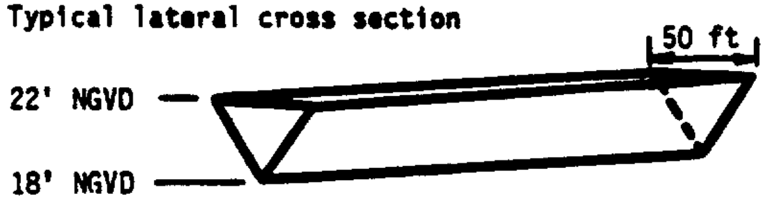
e. SCS curve number, CN

= 1000/(S + 10)
= 1000/(2.4 + 10)
= 81

3. Compute Open Surface Stage versus Storage

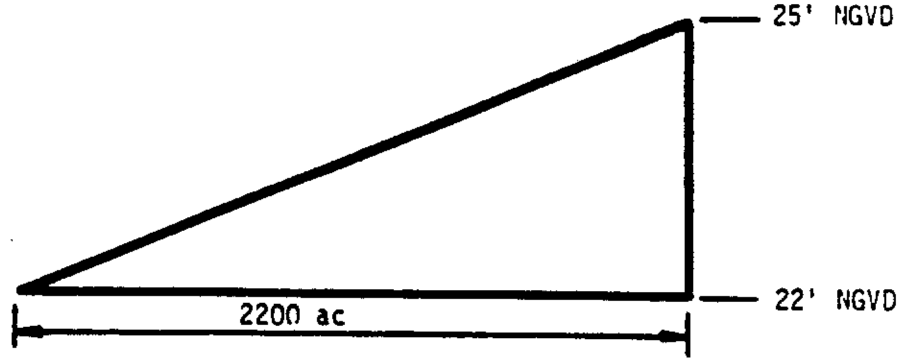
a. Laterals store between elevations 20' and 22' NGVD

Typical lateral cross section



60 acres of laterals at top of bank

b. Developed farm area stores linearly between elevations 22' and 25' NGVD.



	Storage		
Stage (ft,NGVD)	Laterals (ac-ft)	Site (ac-ft)	Total (ac-ft)
20	0	0	0
21	((30 ac + 45 ac)/2)(1 ft)	0	37.5
22	((30 ac + 60 ac)/2)(2 ft)	0	90.0
23	90 + 60 ac(1 ft)	0.5(2200 ac)((1x1 ft)/3)	516.67
24	90 + 60 ac(2 ft)	0.5(2200 ac)((2x2 ft)/3)	1676.67
25	90 + 60 ac(3 ft)	0.5(2200 ac)((3x3 ft)/3)	3570.00

4. Flood Route the Design Storm

   25-year 3-day event

   (See the selected pages titled "Santa Barbara Project Name: Ag Maj Impound" at end of this example)

   Maximum stage in farm (Basin 1) = 23.1' NGVD.

B. Reservoir

1. S = 0.01 in. (District computer program will not accept zero.)

2. Since the reservoir will be used to mitigate for the loss of some wetlands, 1 foot of water will be maintained in parts of the reservoir at elevation 22.0' NGVD. Storage will then start at the control elevation, 23.0' NGVD.

Stage (ft,NGVD)	300-acre Reservoir Storage (ac-ft)
23	0
24	300
25	600
26	900
27	1200
28	1500
29	1800
30	2100
31	2400
32	2700
33	3000
34	3300
35	3600
36	3900
37	4200

3. Allowable Discharge

   81 cfs was determined by pre- versus postdevelopment.

4. Weir crest elevation required

   a. First inch of runoff

   b. Volume = 1 in. x (1 ft/12 in.) x 2560 ac = 213 ac-ft. From the reservoir storage calculation (B.2), the weir crest should be set at elevation 23.7' NGVD, so that 213 ac-ft are detained.

5. Compute weir length

   a. Rainfall from the 25-year 3-day storm is 10.9 inches. Using the S value calculated for the sod farm,

   $$\text{runoff} = Q = \frac{(P - 0.2S)^2}{(P + 0.8S)} = 8.5 \text{ in.}$$

   b. Volume of runoff

   = 8.5 in. x (1 ft/12 in.) x 2560 ac

   = 1813 ac-ft, or a reservoir stage of elevation 29' NGVD (per reservoir stage storage table), or 5.3 ft. above the proposed weir crest.

   c. The maximum design head is 5.3 feet.

   $Q = 3.13LH^{1.5}$

   $L = 81 \text{ cfs}/(3.13 \times (5.3 \text{ ft})^{1.5}) = 2.1 \text{ ft}$

6. Water quality control device should be designed to discharge an amount (Q) equal to 1/2 inch in 24 hours.

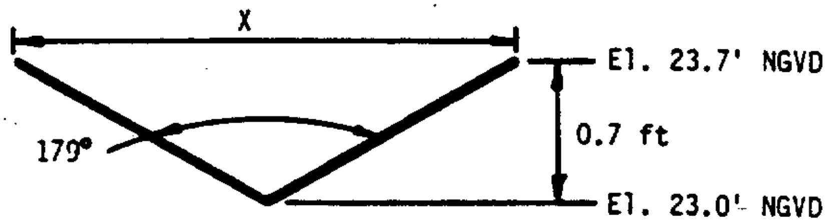
213 ac-ft/2 = 107 ac-ft in 24 hours, or about 54 cfs.
(1 cfs = 1.9835 ac-ft per 24 hrs.)

$Q = 2.5 \times (\tan(\theta/2)) \times (H)^{2.5}$, or

$\theta = 2 \times \tan^{-1}(0.492 \times Q/(H)^{2.5})$

$\theta = 2 \times \tan^{-1}(0.492 \times 107 \text{ ac-ft}/(0.7 \text{ ft})^{2.5})$

$= 179$ degrees



$\tan(\theta/2) = (X/2)/0.7 \text{ ft}$

$X = 2 \times 0.7 \text{ ft} \times (\tan(179°/2))$

X = <u>160 ft</u> - will not fit into weir.

7. Try a triangular orifice

$Q = 4.8 \times A \times (H)^{0.5}$

where Q = discharge, cfs

A = area of orifice, sq ft

H = head above orifice centroid, ft

therefore, Q = 81 cfs [see * on next page]

$A = 0.5 \times b \times h$

assume h = 3 ft

Orifice centroid will be 2/3 of h above orifice invert, or 2.0 ft above elevation 23.0' NGVD, or at elevation 25.0' NGVD.

H = 29.0' NGVD - 25.0' NGVD

= 4.0 ft

$$Q = 4.8 \times A \times H^{0.5}$$

$$= 4.8 \times (0.5 \times b \times h) \times (H)^{0.5}$$

therefore,

$$b = Q/(4.8 \times 0.5 \times h \times (H)^{0.5})$$

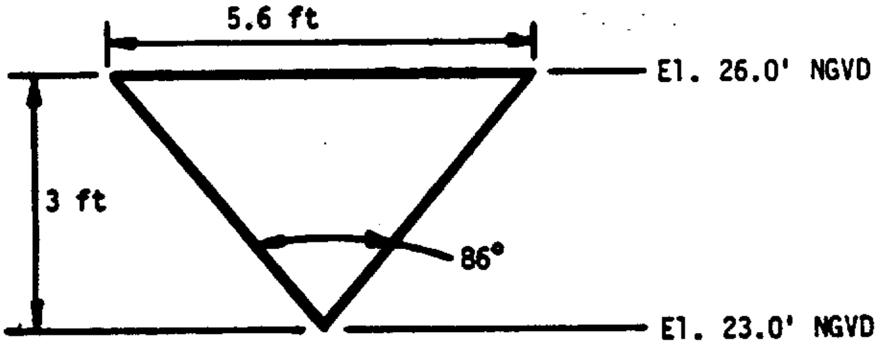
$$= 81\ \text{cfs}/(4.8 \times 0.5 \times 3\ \text{ft} \times (4.0\ \text{ft})^{0.5})$$

$$= 81\ \text{cfs}/14.4$$

$$= 5.6\ \text{ft.}$$

*(NOTE: The orifice was designed using the more restrictive condition created by the allowable discharge criteria rather than the water quality rate.)

Therefore, the structure is 1-3.0 ft high by 5.6 ft wide triangular orifice with an invert at elevation 23.0' NGVD. Only an orifice is used since the allowable discharge is low and the lake size is preferred to be left unchanged, so a V-notch is impractical. The proposed control structure will be considerably smaller than what is required to meet the detention requirements for water quality. Therefore, discharge at elevation 26.0' NGVD will be considerably less than 0.5 inch in 24 hours.



Stage-discharge values for this control structure are given in the computer printouts at the end of this example. The structure should be properly baffled, to reduce the possibility of clogging. The outfall pipe should be large enough so that the peak design storm discharge is not limited by culvert control.

8. Check the routed design storm discharge.

   a. The design storm is the 25-year 3-day event.

   b. The one-day rainfall amount is given to be 8 inches.

c. The three-day rainfall amount

= (one-day amount) x 1.359

= 8 in. x 1.359

= 10.9 inches of rainfall.

d. The pages titled "Santa Barbara Project Name: Ag Maj Impound" also are the selected results of calculations of pumping rainfall runoff into the reservoir (Basin 2), plus the rainfall on the reservoir.

   i. On the last sheet, the peak design storm discharge from the reservoir is listed as 77 cfs. This is deemed acceptable, since the allowed rate of 81 cfs is based on 3/4 in. per day.

   ii. Also on the last sheet, the peak design storm elevation is listed as 28.6' NGVD. District recommended criteria are that if the stored depth of water is greater than 4 feet above the surrounding ground, and berm failure would cause significant damage to the property of others besides the permittee, the impoundment should be designed as a major impoundment.

   The lowest elevations of the surrounding areas are not substantially different from the 22' NGVD on the project, so the 28.6' NGVD stage represents a depth of water substantially in excess of 4 feet; there is a mobile home park just west of the farm. Consequently, this reservoir must be designed as a major impoundment.

   (NOTE: A detailed influence line analysis is beyond the scope of this text. Applicants should seek expert advice from experienced consultants.

   The article "Dam-Breach Flood Wave Models" by Ralph A. Wurbs, in the January 1987 *Journal of Hydraulic Engineering*, Volume 113, Number 1, published by the American Society of Civil Engineers, contains a discussion and comparison of several leading dam-breach flood wave models. Anyone contemplating designing a major impoundment should contact District staff as early in the project development process as possible for guidance on applicable District criteria and methods of analysis.)

   iii. Freeboard requirement will be dependent upon the maximum water depth which is, in turn, dependent upon the design of the overflow structure.

9. Overflow Structure

a. Criteria

i. For gravity filled impoundments, no separate overflow structure is required, because the inflow structure will allow reservoir and farm stages to be the same.

ii. The weir crest should be set no lower than the peak elevation of the routed 25-year 3-day storm.

iii. The minimum freeboard requirement for a given reservoir and depth of water is three feet, or that required to prevent overtopping or failure due to hurricane force winds, as derived from the *South Florida Building Code.*

iv. The weir crest length shall be adequate to pass the peak of:

> the sum of the volume of the appropriate storm (ref. Appendix 6 of *Basis of Review*) falling on the reservoir surface plus the inflow pump hydrograph for the same event, minus the routed discharge control structure outflow.

v. A simple culvert, or culverts, extending through the reservoir side, and with inverts at the design storm peak stage, are not acceptable, because of the danger of erosion of the reservoir berm.

vi. There are at least two acceptable overflow structure designs.

I. A non-adjustable shaft spillway.

II. A properly designed and built non-adjustable sharp- or broad-crested weir in the reservoir berm.

vii. Sodded earthen berms are not acceptable as overflow structures.

viii. Other proposed designs should be discussed with District staff as early in the design process as possible.

b. Design Calculations

i. Treat the system as two basins: Basin 1 (farm) routed into Basin 2 (reservoir).

ii. Basin 1 (farm) shall be considered to have the pumps running continuously.

(Other methods of calculating the pumped flow hydrograph into the reservoir - for the purposes of computing the overflow weir length - may be submitted by the applicant for District staff consideration.)

The pages which begin "Santa Barbara Project Name: Major Impoundment Example, Reservoir" are selected from the hydrograph of continuous farm pumping into the reservoir.

iii. After some preliminary calculations try a <u>100-foot long broad crested weir</u>.

iv. The weir crest should be set no lower than the peak elevation of the 25-year 72-hour routed storm, in this case 28.6' NGVD, to the nearest tenth of a foot.

v. The stage-discharge calculations for the overflow weir are based on the equation $Q = CLH^{1.5}$

where:

Q = discharge, cfs

C = weir coefficient, 2.60

L = weir length, in this case: 100.0 ft

H = head on the weir, ft

vi. Tabulated below are the total stage-discharge values for the reservoir, which are the sum of those previously generated for the control structure plus the overflow structure.

Stage (ft,NGVD)	Control (cfs)	Overflow (cfs)	Total (cfs)
23.0	0.0	0	0.0
24.0	2.6	0	2.6
25.0	14.7	0	14.7
26.0	40.5	0	40.5
27.0	57.2	0	57.2
28.0	70.1	0	70.1
28.6	76.8	0	76.8
29.6	86.8	260.0	346.8
30.6	95.7	735.4	831.1

vii. The pages which begin "Santa Barbara Project Name: Major Impoundment Example, Reservoir" are the results of routing continuous pumped inflow plus the 36-inch 72-hour rainfall into the reservoir, out of the combination of the control and overflow structures. Peak discharge occurs at elevation 29.9' NGVD.

Determining the length and crest elevation of the overflow weir is largely an economic and operational consideration. The length and elevation used in this example result in approximately the minimum additional water depth from the severe storm established in Appendix 6. It is conceivable that additional water depth may be desirable to the permittee. However, it should be noted that additional depth may sharply increase both design requirements imposed by the District and subsequent construction costs.

10. Freeboard Analysis

a. Criteria - Minimum freeboard is three feet above maximum water depth, or that amount required to prevent overtopping or failure due to hurricane force winds as derived from the *South Florida Building Code*, whichever is greater.

b. Minimum crest elevation is at least 32.9' NGVD based on the three foot minimum requirement, but may be higher based on overtopping requirement.

c. The rise in still water depth on the leeward side of the reservoir can be calculated using the Zuider Zee Formula:

$$S = \frac{V^2 \times F}{1400 \times D}$$

Where: V = Wind velocity, mph

F = Fetch, statute miles

D = Average depth along fetch, feet

Wind velocity per *South Florida Building Code* is 120 mph, approximate maximum fetch is about 0.7 mile and average depth is:

$$29.9' \text{ NGVD} - ((22+23)'\text{NGVD}/2) = 7.4 \text{ ft.}$$

Therefore, average increased depth on the leeward side is:

$$S = \frac{(120 \text{ mph})^2 \times 0.7 \text{ mi}}{1400 \times 7.4 \text{ ft}}$$

$$= 0.97 \text{ ft}$$

Maximum still water depth (D_{max}) would be:

$$D_{max} = 7.4 \text{ ft} + 0.97 \text{ ft}$$

$$= 8.37 \text{ ft}$$

Which represents a surface water elevation of 30.87' NGVD.

Maximum wave height is given by the equation:

$$H = 0.034 \times V^{1.06} \times F^{0.47}$$

Where: V = Wind velocity, mph

F = Fetch, statute miles

Therefore:

$$H = 0.034 \times (120 \text{ mph})^{1.06} \times (0.7 \text{ mi})^{0.47}$$

$$= 4.6 \text{ ft}$$

Wave run-up on slope = about 1.5 x H

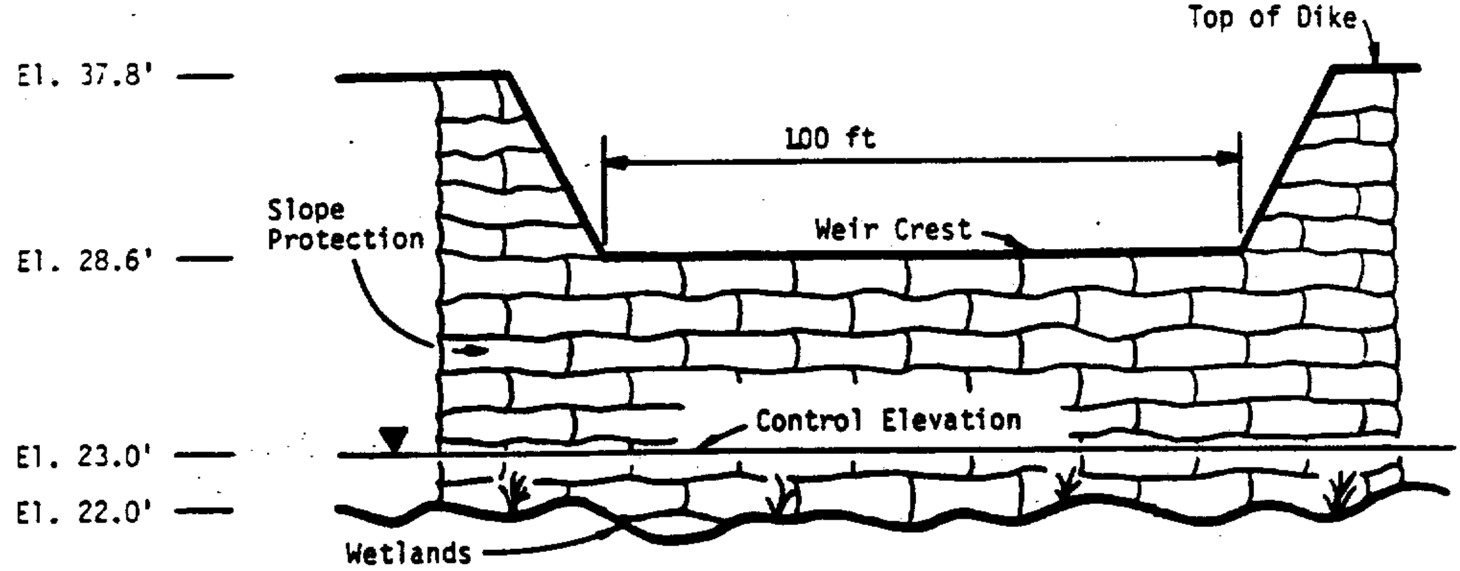
The elevation of the top of the embankment is equal to the maximum still water depth elevation, plus the wave run-up:

= 30.87' NGVD + (1.5 x 4.6 ft)

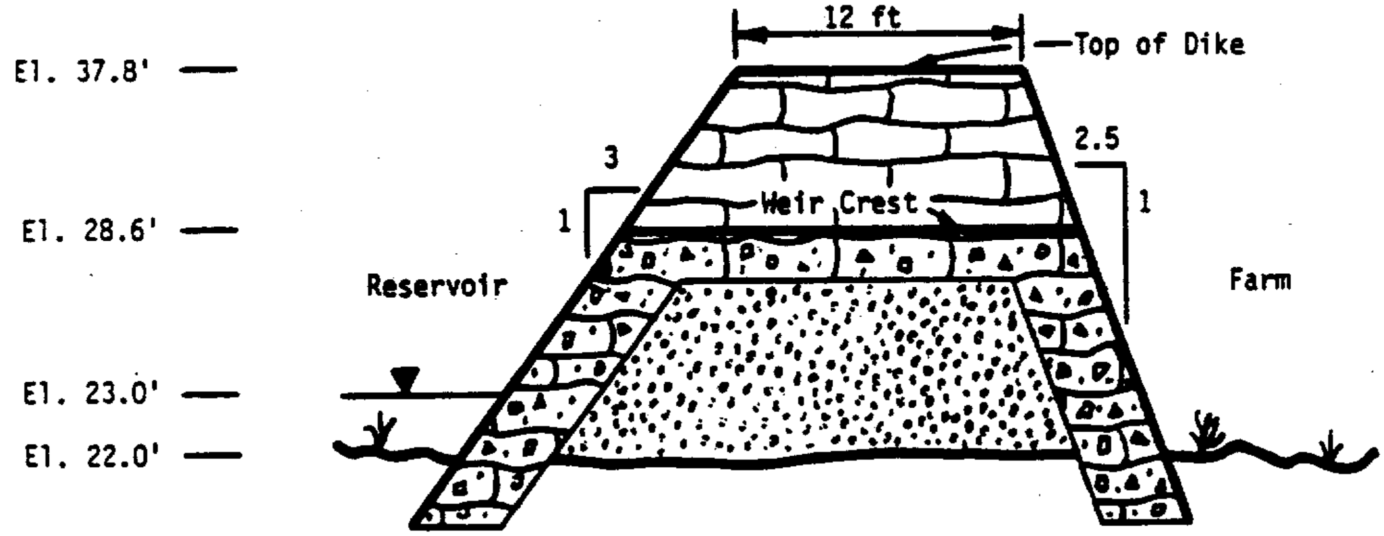
= 37.8' NGVD

Freeboard or wind and wave analysis can become very involved, and a thorough presentation is beyond the scope of this text. The following references are suggested (others may be acceptable): Brater and King, *Handbook of Hydraulics*, Sixth Edition; Saville, McClendon, and Cochran, "Freeboard Allowance for Waves in Inland Reservoirs," *Journal of the Waterways and Harbors Division*, May, 1962; U.S. Bureau of Reclamation, *Design of Small Dams*, 1977.

A sketch of the proposed overflow structure is on the next page. A complete design would include, among other things, proper attention to erosion control around the overflow weir, and energy dissipation devices downstream of it. Additional design requirements may include, but not be limited to, a site-specific geotechnical investigation; and slope stability, seepage and dambreak analyses. Paragraphs 1.4.1 and 2.1.1 - 2.1.5, in Appendix 6 to the *Basis of Review*, also provide a list of design and information requirements for major impoundments.



Elevation View (N.T.S.)



Section View (N.T.S.)

OVERFLOW STRUCTURE

Figure XC-2

SANTA BARBARA

PROJECT NAME . . . . : AG MAJ IMPOUND
REVIEWER . . . . : ED MACIEJKO
PROJECT NUMBER - 2

THE ROUTING IS COMPLETE WHEN THE DISCHARGE FOR BASIN 2 IS REDUCED TO 10.00CFS

********************** BASIN 1 ****************

AREA - 2260.00 ACRES — GROUND STORAGE - 2.40 INCHES

TIME STEP - 1.00 HOURS — TIME OF CONCENTRATION - 4.00 HOURS

RETURN FREQUENCY - 25.00 YEARS — RAINFALL DISTRIBUTION - 3-DAY — 24-HOUR RAINFALL - 8.00 INCHES

STAGE (FT)	STORAGE (AF)
20.00	.00
21.00	37.50
22.00	90.00
23.00	516.67
24.00	1576.67
25.00	3570.00

********************** BASIN 2 ****************

AREA - 300.00 ACRES — GROUND STORAGE - .01 INCHES

TIME STEP - 1.00 HOURS — TIME OF CONCENTRATION - .01 HOURS

RETURN FREQUENCY - 25.00 YEARS — RAINFALL DISTRIBUTION - 3-DAY — 24-HOUR RAINFALL - 8.00 INCHES

STAGE (FT)	STORAGE (AF)
23.00	.00
24.00	300.00
25.00	600.00
26.00	900.00
27.00	1200.00
28.00	1500.00
29.00	1800.00
30.00	2100.00
31.00	2400.00
32.00	2700.00
33.00	3000.00
34.00	3300.00
35.00	3600.00
36.00	3900.00
37.00	4200.00

DISCHARGE STRUCTURE INFORMATION

STRUCT NO.	PIPE SLOPE (%)	DIAMETER (FT)	ROUGHNESS	LENGTH (FT)	WEIR WEIR CREST TYPE/ELEV	WEIR LENGTH	HEAD INVERT ELEVATION	TAIL INVERT ELEVATION

1	SEE PUPP TABLE						
2	.000	****	.100	100.00	NO RISER INCLUDED WITH PIPE		BASIN 2 TO BASIN 3
3	NO PIPE				BROAD/ 28.6	100.00	BASIN 2 TO BASIN 1

BLEEDER INFORMATION

STRUCT NO.	BLEEDER TYPE	DIAMETER OR WIDTH (FT)	ORIFICE INVERT ELEVATION (FT-NGVD)	AREA (FT2)	V-NOTCH ANGLE (DEG)	INVERT ELEVATION (FT-NGVD)	TOP ELEVATION (FT-NGVD)
1	NO BLEEDER INCLUDED IN STRUCTURE						
2	V-NOTCH	NA	NA	NA	86.0	23.0	26.C
3	NO BLEEDER INCLUDED IN STRUCTURE						

PUMP TABLE

PUMP NO.	PUMP ON ELEVATION (FEET)	PUMP OFF ELEVATION (FEET)	PUMP DISCHARGE (GPM)	DISCHARGES FROM BASIN	TO BASIN
1	21.50	20.01	170544.00	1	2

OFFSITE RECEIVING WATER

TIME(HR)	STAGE(FT-NGVD)
.00	20.00
1000.00	20.00

SUMMARY REPORT

TIME (HR)	PIPE NO	BASIN NO	CUMULATIVE RAINFALL (INCHES)	CUMULATIVE RUNOFF (INCHES)	INSTANT. RUNOFF (CFS)	RUNOFF HYDROGRAPH (CFS)	DISCHARGE (CFS)	INSTANT. STAGE (FT)	FROM	TO	STRUCTURAL CONTROL
.0	1	1	.0	.0	.0	.0	.00	20.00	1	2	
	2	2	.0	.0	.0	.0	.00	23.00	2	3	
	3	2	.0	.0	.0	.0	.00	23.00	2	1	
1.0	1	1	.0	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.0	.0	14.3	14.2	.00	23.00	2	3	BLEEDER
	3	2	.0	.0	14.3	14.2	.00	23.00	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
2.0	1	1	.1	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.1	.1	14.6	14.6	.00	23.01	2	3	BLEEDER
	3	2	.1	.1	14.6	14.6	.00	23.01	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
3.0	1	1	.1	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.1	.1	14.7	14.7	.00	23.01	2	3	BLEEDER
	3	2	.1	.1	14.7	14.7	.00	23.01	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
4.0	1	1	.2	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.2	.2	14.7	14.7	.00	23.02	2	3	BLEEDER
	3	2	.2	.2	14.7	14.7	.00	23.02	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
5.0	1	1	.2	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.2	.2	14.7	14.7	.00	23.02	2	3	BLEEDER
	3	2	.2	.2	14.7	14.7	.00	23.02	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
6.0	1	1	.3	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.3	.3	14.7	14.7	.00	23.02	2	3	BLEEDER
	3	2	.3	.3	14.7	14.7	.00	23.02	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
7.0	1	1	.3	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.3	.3	14.7	14.7	.00	23.03	2	3	BLEEDER
	3	2	.3	.3	14.7	14.7	.00	23.03	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
8.0	1	1	.4	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.4	.4	14.7	14.7	.00	23.03	2	3	BLEEDER
	3	2	.4	.4	14.7	14.7	.00	23.03	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
9.0	1	1	.4	.0	.0		.00	20.00	1	2	NO FLOW
		2	.4	.4	14.7		.00	23.04	2	3	BLEEDER
		2	.4	.4	14.7		.00	23.04	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											

TIME (HR)	PIPE NO	BASIN NO	CUMULATIVE RAINFALL (INCHES)	CUMULATIVE RUNOFF (INCHES)	INSTANT. RUNOFF (CFS)	RUNOFF HYDROGRAPH (CFS)	DISCHARGE (CFS)	INSTANT. STAGE (FT)	FROM	TO	STRUCTURE CONTROL
91.0	1	1	10.9	8.4	.0	3.2	380.00	22.28	1	2	PUMP
	2	2	10.9	10.9	.0	.0	70.71	28.07	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.00	28.07	2	1	NO FLOW
OFFSITE STAGE IS		20.00 FT.NGVD									
92.0	1	1	10.9	8.4	.0	2.5	380.00	22.21	1	2	PUMP
	2	2	10.9	10.9	.0	.0	71.68	28.16	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.00	28.16	2	1	NO FLOW
OFFSITE STAGE IS		20.00 FT.NGVD									
93.0	1	1	10.9	8.4	.0	2.0	380.00	22.13	1	2	PUMP
	2	2	10.9	10.9	.0	.0	72.64	28.24	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.00	28.24	2	1	NO FLOW
OFFSITE STAGE IS		20.00 FT.NGVD									
94.0	1	1	10.9	8.4	.0	1.5	380.00	22.06	1	2	PUMP
	2	2	10.9	10.9	.0	.0	73.59	28.33	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.00	28.33	2	1	NO FLOW
OFFSITE STAGE IS		20.00 FT.NGVD									
95.0	1	1	10.9	8.4	.0	1.2	380.00	21.90	1	2	PUMP
	2	2	10.9	10.9	.0	.0	74.52	28.41	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.00	28.41	2	1	NO FLOW
OFFSITE STAGE IS		20.00 FT.NGVD									
96.0	1	1	10.9	8.4	.0	.9	380.00	21.30	1	2	PUMP
	2	2	10.9	10.9	.0	.0	75.43	28.49	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.00	28.49	2	1	NO FLOW
OFFSITE STAGE IS		20.00 FT.NGVD									
97.0	1	1	10.9	8.4	.0	.7	380.00	20.59	1	2	PUMP
	2	2	10.9	10.9	.0	.0	76.33	28.58	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.00	28.58	2	1	NO FLOW
OFFSITE STAGE IS		20.00 FT.NGVD									
PUMP 1 "OFF" AT 97.70 HOURS											
98.0	1	1	10.9	8.4	.0	.6	.00	20.01	1	2	NO FLOW
	2	2	10.9	10.9	.0	.0	76.89	28.63	2	3	BLEEDER
	3	2	10.9	10.9	.0	.1	1.35	28.63	2	1	WEIR
OFFSITE STAGE IS		20.00 FT.NGVD									
99.0	1	1	10.9	8.4	.0	.4	.00	20.01	1	2	NO FLOW
	2	2	10.9	10.9	.0	.0	76.66	28.61	2	3	BLEEDER
	3	2	10.9	10.9	.0	.0	.21	28.61	2	1	WEIR
OFFSITE STAGE IS		20.00 FT.NGVD									

STRUCTURE NO.	PEAK DISCHARGE (CFS)	TIME OF QPEAK	PEAK STAGE (FT-NGVD)	TIME OF HPEAK
1	330.0	33.2	23.1	71.6
2	77.0	97.7	28.6	97.7
3	1.8	97.7	28.6	97.7

BASIN NO.	TOTAL INFLOW (AC-FT)	TOTAL OUTFLOW (AC-FT)	FINAL TIME (HOURS)	FINAL STAGE (FT-NGVD)
1	.13	1589.61	494.57	20.01
2	1589.61	1324.00	494.57	24.79

WEIR LENGTH	100 FT.
WEIR ELEVATION	28.6 FT. NGVD
WEIR COEFFICIENT	2.6
TYPE OF BLEEDER SLOT	TRIANGLAR ORIFICE
SLOT INVERT ELEV.	23 FT. NGVD
ORIFICE HEIGHT	3 FT.
ORIFICE BASE WIDTH	5.6 FT.

	WEIR FLOW IN CFS		
STAGE	WEIR	BLEEDER	TOTAL
23.00	0.00	0.00	0.00
24.00	0.00	2.60	2.60
25.00	0.00	14.68	14.68
26.00	0.00	40.45	40.45
27.00	0.00	57.21	57.21
28.00	0.00	70.07	70.07
28.60	0.00	76.76	76.76
29.60	260.00	86.77	346.77
30.60	735.39	95.73	831.12

S A N T A   B A R B A R A

PROJECT NAME . . . . : MAJOR IMPOUNDMENT EXAMPLE, RESERVOIR
REVIEWER . . . . : ED MACIEJKO
PROJECT NUMBER - 2

THE ROUTING IS COMPLETE WHEN THE DISCHARGE FOR BASIN 2 IS RECUCED TO 50.00CFS

********************** BASIN 1 ****************
AREA - 2260.00 ACRES

GROUND STORAGE - 2.40 INCHES

TIME STEP - 1.00 HOURS

TIME OF CONCENTRATION - 4.00 HOURS

RETURN FREQUENCY - ****** YEARS

RAINFALL DISTRIBUTION - 3-DAY

24-HOUR RAINFALL - 26.50 INCHES

STAGE (FT)	STORAGE (AF)
20.00	.00
21.00	37.50
22.00	90.00
23.00	516.67
24.00	1676.67
25.00	3570.00
26.00	5830.00
27.00	8090.00
28.00	10350.00
29.00	12610.00
30.00	14870.00

XC-20

********************** BASIN 2 ****************
AREA - 300.00 ACRES

GROUND STORAGE - .01 INCHES

TIME STEP - 1.00 HOURS

TIME OF CONCENTRATION - .01 HOURS

RETURN FREQUENCY - ****** YEARS

RAINFALL DISTRIBUTION - 3-DAY

24-HOUR RAINFALL - 25.50 INCHES

STAGE (FT)	STORAGE (AF)
23.00	.00
24.00	300.00
25.00	600.00
26.00	900.00
27.00	1200.00
28.00	1500.00
29.00	1800.00
30.00	2100.00
31.00	2400.00
32.00	2700.00
33.00	3000.00
34.00	3300.00
35.00	3600.00
36.00	3900.00
37.00	4200.00

DISCHARGE STRUCTURE INFORMATION

STRUCT NO.	PIPE SLOPE (%)	DIAMETER (FT)	ROUGHNESS	LENGTH (FT)	WEIR WEIR CREST TYPE/ELEV	WEIR LENGTH	HEAD INVERT ELEVATION	TAIL INVERT ELEVATION	
1	SEE PUMP TABLE								
2	.000	****	.100	100.00	NO RISER INCLUDED WITH PIPE				BASIN 2 TO BASIN 3
3	NO PIPE				BROAD/ 28.6	100.00			BASIN 2 TO BASIN 1

BLEEDER INFORMATION

STRUCT NO.	BLEEDER TYPE	DIAMETER OR WIDTH (FT)	ORIFICE INVERT ELEVATION (FT-NGVD)	AREA (FT2)	V-NOTCH ANGLE (DEG)	V-NOTCH INVERT ELEVATION (FT-NGVD)	TOP ELEVATION (FT-NGVD)
1	NO BLEEDER INCLUDED IN STRUCTURE						
2	V-NOTCH	NA	NA	NA	86.0	23.0	26.0
3	NO BLEEDER INCLUDED IN STRUCTURE						

PUMP TABLE

PUMP NO.	PUMP ON ELEVATION (FEET)	PUMP OFF ELEVATION (FEET)	PUMP DISCHARGE (GPM)	DISCHARGES FROM BASIN	TO BASIN
1	21.50	20.01	170544.00	1	2

OFFSITE RECEIVING WATER

TIME(HR)	STAGE(FT-NGVD)
.00	20.00
1000.00	20.00

SUMMARY REPORT

TIME (HR)	PIPE NO	BASIN NO	CUMULATIVE RAINFALL (INCHES)	CUMULATIVE RUNOFF (INCHES)	INSTANT. RUNOFF (CFS)	RUNOFF HYDROGRAPH (CFS)	DISCHARGE (CFS)	INSTANT. STAGE (FT)	FROM	TO	STRUCTURE CONTROL
.0	1	1	.0	.0	.0	.0	.00	20.00	1	2	
	2	2	.0	.0	.0	.0	.00	23.00	2	3	
	3	2	.0	.0	.0	.0	.00	23.00	2	1	
1.0	1	1	.2	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.2	.1	48.6	48.6	.00	23.01	2	3	BLEEDER
	3	2	.2	.1	48.6	48.6	.00	23.01	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
2.0	1	1	.3	.0	.0	.0	.00	20.00	1	2	NO FLOW
	2	2	.3	.3	48.7	48.7	.00	23.03	2	3	BLEEDER
	3	2	.3	.3	48.7	48.7	.00	23.03	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
3.0	1	1	.5	.0	.7	.0	.00	20.00	1	2	NO FLOW
	2	2	.5	.5	48.7	48.7	.00	23.04	2	3	BLEEDER
	3	2	.5	.5	48.7	48.7	.00	23.04	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
4.0	1	1	.6	.0	45.4	5.5	.00	20.00	1	2	NO FLOW
	2	2	.6	.6	48.8	48.8	.00	23.05	2	3	BLEEDER
	3	2	.6	.6	48.8	48.8	.00	23.05	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
5.0	1	1	.8	.0	82.3	18.7	.00	20.03	1	2	NO FLOW
	2	2	.8	.8	48.8	48.8	.00	23.07	2	3	BLEEDER
	3	2	.8	.8	48.8	48.8	.00	23.07	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
6.0	1	1	1.0	.1	113.3	36.4	.00	20.09	1	2	NO FLOW
	2	2	1.0	1.0	48.8	48.8	.00	23.08	2	3	BLEEDER
	3	2	1.0	1.0	48.8	48.8	.00	23.08	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
7.0	1	1	1.1	.1	139.5	56.5	.00	20.19	1	2	NO FLOW
	2	2	1.1	1.1	48.8	48.8	.01	23.09	2	3	BLEEDER
	3	2	1.1	1.1	48.8	48.8	.00	23.09	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
8.0	1	1	1.3	.2	161.8	77.5	.00	20.34	1	2	NO FLOW
	2	2	1.3	1.3	48.8	48.8	.01	23.11	2	3	BLEEDER
	3	2	1.3	1.3	49.8	48.8	.00	23.11	2	1	NO FLOW
OFFSITE STAGE IS 20.00 FT.NGVD											
[illegible]	1	1	1.5	.1	161.0	[illegible]	.00	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	[illegible]	1.5	[illegible]	48.8	[illegible]	[illegible]	23.12	2	1	[illegible] FLOW
[illegible] IS [illegible]											

STRUCTURE NO.	PEAK DISCHARGE (CFS)	TIME OF QPEAK	PEAK STAGE (FT-NGVD)	TIME OF HPEAK
1	380.0	12.9	25.6	84.1
2	89.7	68.0	29.9	68.0
3	400.9	68.0	29.9	68.0

BASIN NO.	TOTAL INFLOW (AC-FT)	TOTAL OUTFLOW (AC-FT)	FINAL TIME (HOURS)	FINAL STAGE (FT-NGVD)
1	17384.30	23614.44	918.57	21.02
2	23614.44	23452.29	918.57	26.54